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THE DESIGN OF A STEEL STACK

FOR

TRENTON SHOPS

PENNSYLVANIA RAILROAD COMPANY.

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14 June '48. Sec. 9. Univ. M. J. McIlvain

While in the employ of the Pennsylvania Railroad, I designed some power plant stacks. Two of these were of importance. One of the two was to be erected at the new Union Station at Washington, D. C. and the other at the new shops at Trenton, N. J. I have selected the Trenton stack to describe here, although it is smaller and although it does not possess a parabolic bell at the base. My reason in doing this is because at the time of my separation from the Railroad Company, the architects of the Washington station were still trying to get the design, which was of steel, changed to brick. I do not know how this controversy ended, but as the work was not then actually under way, I do not know feel justified in describing something which I did not see started in accordance with the plans that I had laid out.

The design of a stack can be naturally divided into two parts; first, the determination of the height and inside diameter, to conform with the size and conditions of the boiler plant, secondly, the de-

termination of the dimensions of the outside walls and of the lining, so as to carry its own weight, to withstand the force of the elements and to be lasting both without and within. I shall take up these two different features in order.

If there were any good formulae for determining the height and diameter of a stack for given conditions, I was unable to find them. Peclet's method of design uses many assumptions and does not give any corrections for an economizer-loss. I have some interesting curves, compiled by Mr. Albert C. Wood of Philadelphia, which give the draft at the damper of a boiler for various rates of combustion and other assumed conditions, but give no information in regard to the draft required when an economizer is used. The Railroad, therefore, did not want any of these methods used to determine the dimensions of their stack, but wished the losses of draft analyzed one by one. This is scarcely more accurate, perhaps, than Peclet's formula would have been, if allowance had

been made for the economizer, but nevertheless I felt quite safe in undertaking it as I had collected considerable reliable experimental data, which I used to determine all the principal losses, and the references were all backed up by results of many tests, which I had taken great pains to assemble, made by men of known ability.

I herewith submit my report for the height and diameter as presented to the Motive Power Office of the Pennsylvania Railroad Company.

DESIGN OF A CHIMNEY FOR THE TRENTON SHOPS.

It is proposed at Trenton, N. J., to build a power house for the new shops. The power plant will consist of four 400 H.P. Stirling boilers. The stack will be placed to one side of these boilers. Eventually the Company will add on the other side of the stack four more boilers of the same size and style, making a plant of 3200 boiler horse power.

The same stack will be used for all these boilers. The attached prints will show the half of the plant that is to be built now. It is the intention of the Company, that when the boiler plant is entirely built, that it will be of such a size that one or two boilers may be always shut down, and the other six or seven boilers run at rating. It is, therefore, very safe to assume in designing the stack, that the maximum horse power that will ever be required under the most unlooked for conditions will be such that all eight boilers will be fired and running at thirty percent over rating. That is, each boiler will be running at 520 boiler horse power, and the entire plant will be giving 4160 boiler horse power. It is the intention of the Company to burn soft coal under these boilers. It is estimated that they will require about three and three-fourths pounds of coal per boiler horse power. We will assume that each pound of coal will require twenty four pounds of air to burn it or about twice its theoretical protion.

It is almost true that the height of a stack is governed entirely by the draft it is desired to create and the diameter is governed by the volume of flue gases. This, of course, assuming that the temperatures of the flue gases and the outside air remain constant. We shall, therefore, find as well as we can the various losses of draft in the boiler, the flue, the economizer, and the stack itself.

The principal losses of draft are as follows:

Loss No. 1 Due to entrance of air into ash pit,

Loss No. 2 Due to change of direction as air curves to go up through the grate,

Loss No. 3 Due to grate and fire,

Loss No. 4. Due to increasing velocity from being heated and expanded by the fire

Loss No. 5 Due to friction against boiler tubes in the path of gases, etc.

Loss No. 6 Due to decreasing velocity as the gases are cooled by the boiler.

Loss No. 7 Due to turn of gases into the flue,

Loss No. 8 Due to friction of gases in flue,

Loss No. 9 Due to turn of gases in flue,

Loss No.10 Due to friction and bends in path of gases through economizer,

Loss No.11 Due to loss of velocity of gases by cooling and contracting in economizer and flue.

Loss No. 12. Due to turn of gases go up into the stacks,

Loss No 13 Due to friction and cooling in stack.

There are also losses due to change of area of flow. These will be very small, as it is endeavored to keep this area as constant as possible.

LOSS NO. 1.

This loss will be the same in each boiler. We can therefore figure it for one boiler only.

The dimensions of the ash pit door will be about 10' x 2' 5", or 24 square feet. Assume the air in the boiler room to be 80° Fahr. Now running at thirty percent above rating, the number of pounds of air required per hour will be, -

$24 \times \frac{15}{4} \times 520 = 46800$ lbs. The volume of air would be $\frac{46800 \times 53.4 \times 541}{14.7 \times 144} = 638711$ cu.ft. per hour, or 177 cu. ft. per second. The velocity of entrance is therefore $\frac{177}{25}$, or say, 7 ft. per second. From Clark (Rules, Tables & Data, P. 891) we have for the flow of air through an orifice,

$V = 352 \times .56 \sqrt[2]{(1 + .00203\ (t - 32))\ h/p}$, where V = velocity of flow in feet per second. t = temperature in degrees Fahrenheit. h = water gauge to cause flow p = barometric pressure in inches, By substituting for p, 29.92 and for t, the temperature of the fire room, 80° F., and reducing we have

$V = 352 \times .56 \sqrt[2]{1.09744 \times h/29.92}$ or $V = 37.74 \sqrt[2]{h}$

Therefore $h = \frac{V^2}{(37.74)^2} = \frac{49}{(37.74)^2} = .034$ inches of water.

LOSS NO. 2.

I shall assume that in turning, the loss is equal to the energy of motion the air has at entrance. I know the loss will not be more than this, and as it is a very small quantity at most, so I think that this is a close enough approximation.

$$\frac{V^2}{2g} = \text{the loss} = \frac{49}{64.4} \text{ feet of gas head.}$$

Now the volume of one pound of air at 80° F. and at atmospheric pressure is,

$$\frac{53.4 \times 541}{14.7 \times 144} = 13.65 \text{ Cu. Ft.}$$

Therefore, the loss No. 2 in inches of water is

$$\frac{49 \times 12}{64.4 \times 13.65 \times 62.5} = .0107".$$

LOSS NO. 3.

This loss will depend greatly upon the kind of grate or stoker used and its area, which will govern the thickness of the fire. I have decided to figure on .370" of water for this loss, as it is not decided what kind of a stoker will be used. This value is taken from a test made by Mr. William Downs on the West Virginia Pulp & Paper Company's boiler plant. The West Virginia Pulp & Paper Company's boiler plant has some features in common with the proposed Trenton plant. According to Mr. J. M. Whitham, (Trans. A. S. M. E. Vol. XVIII), .370" drop over the fire with common grates will burn over forty pounds of coal per square foot of surface per

hour. From an average of five tests made by myself on the Babcock & Wilcox stoker under a Babcock & Wilcox semi-marine boiler, this draft will burn 28.8 pounds of the kind of coal we intend to use at Trenton. The stoker that will be put in at Trenton will have at least as much air space as the Babcock and Wilcox stoker. Therefore, I think that the value of .370 of an inch is quite ample for this purpose. As a matter of fact loss 3, as measured includes losses 1 and 2. I have, however, put losses 1 and 2 in separately more for the sake of a complete analysis than for a desire of accuracy. Losses 1 and 2 are so small as not to make very much difference in the final height of the stack required.

LOSSES NOS. 4, 5, 6 & 7.

All these losses may be summed up together and called .23 of an inch. This figure is again taken from Mr. Wm. Downs' test, and is I think ample.

LOSS NO. 8.

I spoke above regarding the area of the flues,

etc, being constant for each boiler. As the flue itself is of constant area, this does not seem to be true. The products of combustion will be made, however, to enter the flue in such a way that a portion of its area in that part which is not carrying the gases from all the boilers will be dead space with no air current in it. We will, therefore, take the total length of the flue from the farthest boiler, and the velocity of gases as if all the boilers were discharging into the flue at the farthest point from the stack, and figure out the loss due to friction in that way. This assumption is slightly inaccurate, but is more accurate than the formula itself, which we will use.

Assume the flue temperature to be 572° F. Then the volume of the products of combustion of 24 pounds of air and one pound of coal is 628 cu. ft. (See "Helios", page 14). The total volume of the gases for the four boilers running at thirty percent over rating per second is, therefore,

$$\frac{520}{60 \times 60} \times \frac{15}{4} \times 628 \times 4 = 1361 \text{ Cu. Ft.}$$

The flue is 5' 10" x 7' 4" = 42.73 Sq. ft.

Therefore, the velocity is 1361 divided by 42.73 or 31.85 feet per second. The length of the flue is 63' 0". The perimeter of the flue is 26.33 feet.

Therefore, the water gauge due to friction is,

$$\frac{.000000001872 \times 63 \times 26.33 \times (31.85)^2 \times (60)^2}{5.2 \times 42.73} = .05104"$$

Where .000000001872 is the coefficient of friction as given by Navier (See Coupilliere's Course d' Exploitation des Mines, Volume 2, Page 389).

LOSS NO. 9.

It is the intention not to make the turn in the flue with sharp corners, but to make it by gentle curves. It is, I think, therefore, very safe to omit this value as it will be very small indeed, and as a matter of fact, it is taken care of by the coefficient used in the formula above.

LOSSES NOS. 10 & 11.

It is the intention that the arrangement of the economizers at Trenton will be three economizers in

series, each 16 tubes long and 8 tubes wide, on a ten tube spacing. The Manhattan Railway Power Station has exactly the same arrangement of boilers, flues and economizers as we propose to use at Trenton except that their boilers are 520 H.P. each or thirty percent larger than our boilers, and that they have four economizers in series instead of three. Now in a trial by Mr. H. D. Thomlinson on August 19th, 1903, it was found that when the boilers were running at thirteen percent over rating, the loss of draft in the economizer was .23 of an inch (See Trans. New York Railroad Club, Volume XIII, Page 398). Our economizer being three quarters as long ought to require only three-quarters as much draft when the boilers are running about forty-five percent above rating, which corresponds to the boiler horse power the Manhattan plant was developing. I shall take, however the figure .23 of an inch, the same as the Manhattan plant, although it is a little larger than necessary, because this one test is the only test

that I know of where the arrangement of boilers, etc, is the same as the proposed Trenton plant, and I do not want to be absolutely sure of a reading taken only at one test. The Green people give the formula for the loss of draft due to economizers as

$$L = \frac{(H)^2}{(3910)^2} \times \frac{n}{64} \times .225$$

Where L equals loss of draft in inches of water, n equals number of tubes of length of economizer, H equals number of pounds of air per square foot of open economizer area per hour. It is easy to see how this formula was dreived by assuming certain laws about friction which may or may not be true, and which certainly are not true between very wide limits. These constants were obtained from a plant where each boiler had its separate economizer. Applying this formula to our case we have,

$$\frac{(520 \times 15 \times 24 \times 4)^2}{(4 \times 39.75 \times 3910)^2} \times \frac{48}{64} \times .225 = .2442''$$

I think, however, on account of the high velocity of the products of combustion in our problem that this result is too large, and I prefer to take the figure

found at the Manhattan plant.

LOSS NO. 12.

This loss I think will be very small, but I do not know how to calculate it. From observations that I have made it is not possible as a rule, to notice with an ordinary water gauge the difference in pressure on both sides of a bend in a flue or in the flue near the stack and in the base of the stack just above the flue. I shall, therefore, call this loss equal to .03" of water, which is a quantity that can be read on a water gauge, and is therefore, larger than the actual loss sustained. Summing up all these losses so far we have :-

Loss No. 1	.034	inches	of	water
" " 2	.011	"	"	"
" " 3	.370	"	"	"
" " 4, 5, 6, & 7	.230	"	"	"
" " 8	.051	"	"	"
" " 9	.000	"	"	"
" " 10 & 11	.230	"	"	"

Loss No. 12 .030 inches of water

Total Loss .956 " " "

Let us investigate the draft in a stack 225 feet high, which is practically the same thing as 225 feet above the grates. Let us assume that the outside air will reach 90° F. in summer. Let us assume that the temperature in the stack is of an average of 375° F. Then the water gauge that such a stack closed would create is

$$\left(\frac{1}{551} - \frac{1}{836}\right) \frac{225 \times 493 \times .0807}{5.2} = 1.06''$$

The height of the stack seems, therefore, as if it would do nicely, although we have not yet calculated on the friction in it.

Let A be the area of the required chimney at top.

Total weight of coal burned per hour is,

$$\frac{15}{4} \times 8 \quad \times 520 \quad = \quad 15600.$$

$$\frac{15600}{A} = (2\sqrt[2]{225} - 1) \times 8 \quad \therefore \quad A = 67.2 \text{ Sq.Ft.}$$

(This is the formula used by Mr. H. R. Heinicke, a radical brick stack builder)

$$A = \frac{.06 \times 15600}{\sqrt[2]{225}} = 62.4$$

(Kents Formula)

Let us call the stack 9' 6" in diameter at the top.

In regard to loss No. 13 which I have, or rather had to until now leave out, Mr. Heinicke's formula is supposed to be correct for this friction. It is customary with Kent's formula to add a strip two inches wide all the way around the stack for dead air space, or one may assume a height for the stack and use such a formula as I used to calculate the loss due to friction in the flue. This formula I would not want to use for three reasons: First - Probably the gases cool very much and, therefore, contract and change their volume, Second, - The area is not constant on account of the taper of the stack, and Third, - This formula is arranged for low velocities, and gives too high values when the velocity increases very much.

In regard to adding a strip two inches wide all around the stack, it would only be necessary to increase the diameter at the top as far as draft goes, for this stack will have a taper. If we add this

two inches to the area found by Kent's formula, we arrive about at the same value that Mr. Heinicke gives. That is very close to 9' 0".

We will, therefore, call the stack 225' 0" high and 9' 0" inside diameter at the top.

Respectfully submitted,

John M. Maris.

Altoona, Penna.,

18th July, 1904.

We may sum up all these losses of draft in a general way as follows:

Loss due to fire and grate	.415
Loss due to boiler	.230
Loss due to economizer,	.230
Loss due to flue	.081
Total	.956

This, of course, assumes that the stack is made of such a diameter as practically to eliminate friction in its length.

The next question to be decided is the style of a stack to be used. The Pennsylvania Railroad Company was a large buyer of the Custodis stack in former years. However, it has had bad luck with these purchases. The stacks at the Olean and Renovo shops both cracked slightly. The stack at the West Philadelphia shop cracked and had to be banded. The two stacks at the Altoona machine shop and at the Altoona car shop both cracked so badly that they had to be taken down. A report was made by Mr. Jay M. Whitham representing the Custodis people, and the Assistant Engineer of the P. R. R. on the cause of these failures. This report seemed to show that the cracks were due in all cases to over-heating. At all these plants shavings were sometimes used under the boilers. These shavings were carried over by the draft to the base of the stack where they then burned. Occasionally explosions were heard in the up-take. The temperatures of the stack, whenever taken, were found to be higher than could be read on a 1,000 ° F. pyro-

meter. In one instance the asphaltum paint was burned off the cleaning door in the base of the stack some feet below where the flue gases entered. All these things led to the conclusions that the stacks which were unlined and built for 600° F had been abused and greatly over-heated.

However, the Railroad seemed to lose faith in radial brick stack construction. The Webber, Kellog and Steinl stacks were also examined, and I myself after inspecting quite a number made a report on the Heinicke stack. The road did not seem however to find in any of these stacks a marked advantage over the Custodis. It was, therefore, decided to use a steel stack.

The objection to a steel stack, besides its lack of beauty, was its eating away from the inside, due to sulphur fumes and perhaps steam. An iron stack was, of course, almost impossible to get, and I doubt if it would have been much better. In the report made by the Westinghouse, Church, Kerr & Company to the Pennsylvania Railroad upon their new power plant in

Long Island, they recommended to prevent corroding on the inside that the stacks there be made of steel and lined with fire brick grouted behind with cement. This lining is to extend to the top of the stack. I was accordingly ordered to build the Trenton stack of steel, and line it to the top with fire brick as was being done in New York.

We now have to consider the design of the stack with respect to the thickness of its plates, etc. In the first place the Railroad has several standards in regard to stacks that it was necessary to follow. They have a standard astrigal, which is, I think, a good one. This was, therefore, copied. They always in stack building, put the upper plate inside the lower, and make each shift of plates a true cylinder. This is better than the sloping method sometimes used, because the stack is not then streaked by the rain, is easier to build and is stronger near the base. The Road has also a standard anchor plate, anchor bolt, and stack base so these things were like-

wise adopted.

I was in a quandary at first to know how to find the thickness of plates required, until I got two ideas from Westinghouse, Church, Kerr & Company by looking over the design of one of their stacks. These suggestions were to plat the stress called out per inches circunference of the iron, and to make the allowable stress per square inch 1,000 pounds for every sixteenth of an inch of thickness of the plates; the remainder of the design I worked out myself. The reason for platting per inch of circumference will be apparent after looking at the curves. The reason for assuming the strength of the plate per square inch to vary as the thickness I will explain. Now if we have a one sixteenth inch plate, the allowable stress per square inch is 1,000 pounds. If we have a one inch plate the allowable stress per square inch will be 16,000 pounds. When I first came to consider the stresses in a stack I was very much perplexed. A stack is subjected to compression

due to its own weight, something in the manner of a column due to a superimposed load. It is also subjected to bending, due to the wind. A little consideration, or a glance at the stress curves, will show that the stack will fail at the compression side before it will tear on the tension side. I decided that I would not consider a stack as a column for this reason. It can only fail in two places, assuming on very good authority that it will not bulge as a whole. The rivets may either shear or crush at the joints, or the plate may crinkle between two joints. As the plate is double at the joints this really divides the stack into short columns running from joint to joint, and if the rivets are properly placed we have only to fear the crinkling of the plates. This is what led Westinghouse-Church-Kerr and Company to consider the strength of the plate per inch of circumference to very as the square of the thickness of the plate. This bears an analogy to the strength of the column per square inch, which varies

almost as the square of the radius of gyration. (See Berg's Safe Building, Page 25). I do not know what theoretical reasons except what I have just mentioned led them to make this rule for compressive strength of stacks, but I do know that they have tried it out many times, and found it always safe and yet not wasteful of material. All column formulae are built on assumptions, and must be proven by actual tests. Therefore, I think that I am justified in using the above rule, which is probably quite accurate, and I feel much more accurate than most stack formulae. My next trouble was this. If I take the stress due to the weight of the material, how am I to find the stress due to the wind moment separately. It does not seem that I ought to take the moment of inertia for the formula M equals S x I divided by C, about the center of gravity of the stack at any given section, because the center of gravity is not on a neutral axis, on account of the direct compression. In the first stack I designed

I corrected for this error by using the formula given by Robertson in his "Strength of Bridge Members." After I finished this job, I worked out the maximum stress over again by adding the compressive stress to the stress called out by the resisting moment, assuming the neutral axis to pass through the center of gravity of the section. I found that my error in the last calculation was so slight that it was not worth while to take all the trouble I had spent in correcting for it, and therefore I used this same approximation for all future stack problems including this one.

I will now derive my formula for the stress in in the stack due to the wind at any section. Assume that on account of the plates lapping eachother on the way down the stack is conical.

Let W = the wind pressure per square foot of projected area of stack = 25 pounds.

Let l' = outside diameter at top in inches.

Let h = distance from top of stack to section under

consideration in feet.

Let D = the outside diameter of stack at section under consideration in inches.

Let d = the inside diameter of shell at section under consideration in inches.

Let t = thickness of steel at section under consideration in inches.

Let q = stress per inch of circumference in pounds

Let V = mean circumference of section under consideration in inches.

Let S = stress called out per square inch. in pounds.

Let I = moment of inertia of the section about a line taken through its center of gravity in inches.

Let M = the moment in inch pounds due to the wind = the moment in inch pounds called out in the section.

$$M = \frac{W\,l'}{12} \times h \times \frac{h}{2} \times 12 + \frac{(D - l')W}{12} \times h \times \frac{1}{2} \times \frac{h}{3} \times 12$$

$$= \frac{W\,h^2}{3}\left(l' + \frac{1}{2}D\right)$$

$$M = \frac{2S\,I}{D} = \frac{S(D^4 - d^4) \times 3.1416}{32D}$$

t S = q, it is S = q divided by t.

$$M = \frac{q(D^4 - d^4)\,3.1416}{32\,D\,t} = \frac{q\,(D^2 + d^2)\,(D^2 - d^2)3.1416}{4 \times 8\,D\,t.}$$

$$\frac{(D^2 - d^2)3.1416}{4} = V\ t \qquad \therefore V = \frac{(D^2 - d^2)3.1416}{4\ t}$$

$$\text{It is. } \frac{q(D^2 + d^2)V}{8\ D} = \frac{W\ h^2}{3}(l' + 1/2D)$$

$$\therefore \quad q = \frac{8D\ W\ h^2(l' + 1/2D)}{V(D + d)3}$$

Where W = 25 pounds.

It will be seen that q is independent of t which is correct. I have worked this equation out in other forms, but I find the one given the most convenient; $D^2 d^2$ and V can be looked up in the tables of any hand-book.

It will be seen that they are three separate stress curves. The one due to the weight of the plates (the rivet heads and laps and ladder being neglected) to which is added the weight of the astrigal, the one due to the weight of the lining with its supporting Z bars, and the one due to the wind. All these three curves, also the summation curve and also the allowable strength of plate curve must be plotted at the same time. The curve for the strength of riveted joints may be put in afterwards. However we must start one of these curves first, as for in-

stance the curve of stress due to the weight of the plate. It will be noticed that the curve sheet is sectioned. The abscissae represent the height of the stack with the top at the left of the figure, where, of course, the curves are started. The vertical lines are drawn every five feet since it was decided to make the horizontal joints in the stack five feet apart. The ordinates represent the stress per inch of circumference, and are divided for every hundred pounds.

In plotting the curves first we assume the thickness of the top plates, which in the case at hand was taken as 5/16" in good practice. Now to commence the plate stress curve, the astrigal, by computation, from the accepted design weighs 10 1/2 pounds per inch of circumference at the top of the stack. This weight is laid off on the zero line of height. The weight of the plate is found from any table to be 1.06 pounds for a strip one inch wide and one foot long, hence the weight of plate at ten feet down the

stack per inch of circumference is 10.6 pounds. If we add to this weight the weight of the astrigal, per inch of circumference, we find the height of the plate curve at the vertical line representing ten feet to be 21.1 pounds. Of course, one may say that as the diameter of the stack is increasing all the time the plate curve should be corrected for this enlargement. This is perhaps true, but as the weight of the laps, the rivet heads, the ladder and the paint are omitted, I think that this curve is nearer correct than it would at first appear. For a very complete analysis, it might be well to figure the weight of the stack carefully to different sections, and divide by the circumference in inches at that section, but where work has to be got out quickly, and since the stress due to the weight of the plate is such a small factor at best when compared with the wind curve, the method that I followed is plenty accurate enough. If the stack had a bell at the base it would be necessary on account of the great increase in diam-

eter to make the necessary correction for the increased circumference by getting the total weight to the section in question as stated above. I want to settle a little criticism that may arise concerning all the three stress curves right here. That is the curves ought to have little breaks in them every five feet where the section suddenly changes. In all cases where the strain is figured at the joint, it was fugured for the top or weaker plate at that joint, and the curve drawn through these points calculated will be consequently higher and therefore show a greater strain than an accurate analysis would show. As this paper is not supposed to correct for errors that are not of any importance but is supposed to show a method of designing a steel stack against time, these little corrections are not made in any of the curves. In dismissing the plate curve, we would say that this curve is carried on together with the other curves to eighty feet from the top, where it is found necessary to use a heavier

plate. At this point the weight of plate per inch of circumference is 80 x 1.06 + 10.5 = 84.8 + 10.5 = 95.3 pounds. At this point, as we have said, it is found necessary to use a 3/8" plate which weighs, ina strip one inch wide and one foot long 1.28 pounds. Therefore, at ninety feet from the top, the stress curve would be 95.3 + 1.28 x 10 = 108.1 pounds. The plate curve is platted in this manner with the other curves all the way down the stack.

We now pass on to the curve due to the weight of the lining. The lining is supported with six inch Z bars every twenty feet. These Zee bars are held to the stack by the same rivets that hold the plates at the joints when the joints are single riveted. When the joints are double riveted, the Z bars are fastened just below, as it is not possible to put two rows of rivets through the Z bar flange. The first wieght that the plate receives from the lining is therefore twenty feet below the top of the stack. The weight of the lining is taken at .08 pounds per

cubic inch. Now at twenty feet from the top the inside diameter of the stack, just above the joint, is 120 5/8 inches. Inside the lining at the same point, the diameter is 108 5/8 inches. Therefore, the mean diameter is 114 5/8 inches and the mean circumference is 360.11 inches. If we approximate at all to the weight of the lining, we must, to be safe, make the lining appear a little heavier than it really is, and this is what I propose to do. Therefore, for the weight of the lining for the first twenty feet we will have, .08 x 360.11 x 6 x (20 x 12) or 41,484.67 pounds. Now the Z bar weighs 22.7 pounds per foot. Therefore, the total weight of the Z bar is 22.7 x 360.11 ÷ 12 = 681.21 pounds. Therefore the total weight supported by the Z bar at twenty feet is the sum of 41,484.67 and 681.21 or 42,165 pounds. The mean diameter at the section in question is about 120.82 inches. Therefore the mean circumference is 379.57 inches. The load per inch of circumference is consequently 42,165.88

divided by 379.57 or 111 pounds, which is the height of the curve due to the weight of the lining at twenty feet from the top of the stack. Now at forty feet the mean circumference of the lining is found as before, and this is used to find the weight of the lining and Z bar for its support between section twenty and forty. To this is added the weight of the first twenty feet, namely 42,155.88 pounds and this sum is divided by the mean circumference of the stack at section forty. The same operation is performed at sixty feet and eighty feet and so on down the stack, always adding the weight of the lining already found to the weight of the lining for the first twenty feet above the section in question. At a little less than forty-five feet from the base of the stack is the last Z bar. The lining from here on is carried on the foundation. The curve from this last Z bar is considered a straight line. It should really drop a little due to the increasing of the stack's circumference, but as there is no regular bell on the base

we will assume as we did with the weight of plate curve that the stack is a true cylinder. The lining at the base is spread out to twelve inches. The pressure per square inch on the lining at the bottom due to its own weight is, therefore, about 45 x 12 x 9 divided by 12 or 405 pounds. This is well within the crushing strength of fire brick. Over the openings for the flues there is a Z bar, put there more to stiffen the openings than to support the lining . As the lining rests on the foundation, it will really arch itself over these openings and the thrust in the arches will be taken up by a horizontal tension in the plates. Therefore, the stress the plates receive from the weight of the lining at the flue openings has been disregarded. In Baker's Masonry, a method is given for finding the bending moment in lintels, etc., caused from brick work above. This moment is very small compared with the total moment all the lining above the opening would create, and, therefore, in my problem, as I have said, the

firebrick is not considered as resting on the Z bars above the flue openings at all.

We will now say a word in regard to the curve due to the wind. I have before worked out the formula due to the stress caused by the pressure of the wind, which is tension on one side of the stack and compression on the other. The wind load curve is platted from the results of this formula. When the curve is platted as far as the roof line a question arises. There will be no wind pressure below this line, therefore, shall we not make the curve from here on a tangent line? As the stack was going to be built before the boiler house, and as this extra wind pressure amounted to very little at most, from this point on, as will be seen from the curve diagram, which shows almost a straight line at this point, it was decided to figure the wind as acting all the way down the stack.

Now the ordinates of the curves that we have found, if added together, will give the maximum com-

pression that will occur in a stack per inch of circumference. This curve is also platted on the curve sheet. The sum of the ordinates of the plate curve and the lining curve, subtracted from the wind curve would give the maximum tension in the plates per inch of circumference. This curve was originally platted by me on the curve sheets of stacks. I found, however, that when a stack is lined, the tension was so small as not to cut any figure compared with the compression, and it was only necessary to not get the pitch of the rivets so small that the joints in the stack would not stand much tension It is almost impossible, however, to make a mistake here, because the pitch of the rivets is limited by the size of the rivethead, for it is necessary to leave room to drive them properly. Therefore, on my subsequent stack curves, including this one, the tension is entirely omitted.

It is now necessary to plat the curve due to the strength of plate. I have discussed the three stress

curves as though each were drawn to completion before the thickness in the plates was determined. A moments consideration will show, however, that this method was impossible, and that it was necessary to plat all curves practically at once.without letting any one curve get far ahead of the others, because where it it is necessary to change the thickness of the plates, it was also necessary to change the other curves which depend on it. The first thickness of plate, as I have said before was chosen as five-sixteenths of an inch. The stress allowed in this plate per square inch was therefore 5 x 1,000 or 5,000 pounds. Now above the roof line the stack was subject to the effects of the weather. It was, therefore, decided that above the roof only eighty per centum of the allowable stress would be used. Eighty percentum of 5,000 pounds is 4,000 pounds, which is the stress per square inch that we will allow on the top shift of plates. The stress allowed per inch of circumference is, therefore, five sixteenths of fourthousand

pounds or twelve hundred and fifty pounds. This curve is platted, as can be seen from the diagram to to this height. It will be noticed that the weight of the lining added at eighty feet from the top of the stack is too much for a five-sixteenth inch plate. At this point, therefore, the next heaviest or three-eights inch plate must be used. The height of the curve at this point is, therefore, 3/8 x 6,000 x .8 or 1,800 pounds. The plate has to again be made thicker at one hundred feet from the top of the stack and so on down to the roof line of the building. At this point the allowable compression in the plate is 10,000 x 5/8 x .8 or 5,000 pounds. From here on there is considered to be no rusting effect, and the full working strength of the plate is allowed, namely, 10,000 x 5/8 or 6,250 pounds per square inch of circumference as shown on the curve sheet. It will be seen that the summation curve of the three loads on the stack reaches about 5,850 pounds. If the stack had been much longer, or if it had been entirely out of

doors, it would have been economy to use a bell. This bell in practice is generally not parabolic, but each plate is put on a batter of about 3/8" to the foot, besides which there is, of course, the extra spread due to the plates lapping one another. The heaviest plates used in our stack were 5/8" in thickness. This weight of plate had to be put on about sixty feet above the foundations. It would not have been practical to have started a bell up so high that the size smaller, namely, a 9/16" plate could have been used on account of the large diameter of the stack at the bottom, and on account of the increased expense. This large diameter would not only have complicated the flue openings, but would also have crowded the boiler house very much.

In regard to the opening in the stack for the two flues almost nothing has been so far said about them. It is only necessary to substitute a section of metal equal to that cut out. This has been done as will be seen from the drawing by putting structur-

steel shapes and plates along the sides of area equal to the metal taken out. This reinforcement is riveted to the stack securely, so that the rivets will not shear or crush before the allowable stress in the plates is reached.

We now come to the rivet curve in our curve diagram. This may be platted after the other curves are all finished, as it is always possible to make the joints strong enough. As I have said before, the joint has to be designed only against compression. Now the only way the joints can fail is by crushing or by shearing the rivets. All the joints are so arranged that the rivets will shear before they will crush. This result can easily be obtained by looking up the strength of rivets in any steel company's hand book. These show very clearly the size of rivets for certain thicknesses of plate that will shear before they will crush, and it is only necessary to choose such a size rivet in designing a joint.

The first size of rivets used are 3/4 inch on a 2 1/2 inch pitch. The area of the rivet is consequently .442 square inches. The safe shearing stress per square inch is assumed to be 10,000 pounds. Therefore, the safe shearing value of each rivet is 4,420 pounds. This rivet supports 2 1/2 inches of plate. The allowable rivet stress per inch of circumference is then 4,420 x 2/5 or 1,768 pounds. As in the allowable stress of plate curve, only 80 percent of this value is used above the roof line to allow for rusting. Eighty percent of 1,768 is 1,414.4 pounds. This will be found to be the value that is platted on the curve sheet. This size rivet will be found strong enough until we get down eighty feet from the top of the stack, where 7/8 inch rivets on a 2 1/2 inch pitch are used. The same method of calculating the strength of the rivets per inch of circumference is used as before, and we go on using stronger and stronger joints successively until we get 55 feet from the base of the

stack. Here it is necessary to use three rows of 7/8 inch rivets, two of which are on a 2 1/2 inch pitch and one on a five inch pitch. The allowable strength of the joint per inch of circumference is, therefore, .601 x 10,000 x .8 = 4,408 pounds, since there is just one rivet for every inch of circumference. When we get below the roof, however, we allow the full working strength of the rivets, just as we did with the plates. This is .601 x 10,000 or 6,010 pounds per inch of circumference as can be seen from the curves.

Now in regard to the vertical seams in the plates, nothing has been said about them before. According to our formula for strength of plates, no stress is created in these joints at all. I do not think this is very f ar from true, because we use such heavy plate, and such a heavy wind pressure that the sheets are not liable to have a tendency to buckle. These seams are, therefore, taken from practice without any calculation whatever.

The rivets were put on a three inch pitch all the way down the stack. Three-quarter inch rivets were used for the first 80 feet, and 7/8 inch rivets were used all the remainder of the way down, as will be seen from the drawing.

We next come to the anchorage of the stack. As the sheets rest on a cast iron base, the rivets which hold the sheets to the base plate need stand only with shear due to tension. Now this tension per inch of circumference is the difference between the height of the wind curve and the sum of the weight of plate and lining curves. From the diagram this is found to be about 3,150 pounds per inch of circumference. On a three inch pitch with 7/8 inch rivets, the efficiency of the plate against tearing is say 66 percent, allowing for the injured plate around the rivet hole. If the allowable stress against tearing is taken at 16,000 pounds per square inch, the safe tension per inch of circumference on the plates will be 16,000 pounds x

x 5/8 x 2/3 = 6,667 pounds, which is more than what is required. In regard to the rivets, the same method is used in finding the strength of that joint for tension as was used farther up the stack for compression, always being sure, however, that the rivets will not crush. In the given conditions and dimensions, this strength per inch of circumference is, .601 x 10,000 x 2/3 or 4,007 pounds, which is safe.

We now come to the foundation bolts, they also have to stand only tension. To find this strain exactly would be a very difficult task. I therefore propose to approximate as follows. The circumference of the stack at the base is 501.41". There are sixteen holding down bolts, therefore, each bolt must have to take care of one-sixteenth of the circumference or 31.34 inches. Now for each inch of circumference there is a pull of 3,150 pounds, as we know. Therefore, the total tension on each bolt is 3,150 x 31.34 or 98,720 pounds. Of course, there are many objections to the method of calculations

I have used. It must be remembered, however, that on one side the stack is pressing down over a certain area of concrets, and on the other it is pulling up against certain given comparatively small rods of steel and, therefore, an exact solution would be I think very difficult to obtain. I consequently think that my solution of the tension in foundation bolts, although probably too large, is close enough for practical purposes. The Road has its standard size of anchor bolts, namely 2 1/2 inches, which it was necessary to use. I would have preferred to use a larger bolt, but I think that the size used will hold all right. Say we figure on an allowable stress of 20,000 pounds per square inch. I have taken this allowable stress a little high because the method I used to find the tension in the bolts gives values somewhat high. Now the area of such a bolt at the root of the thread is 3.64 square inches. Therefore, the allowable tension in the bolt is 72,800 pounds. The tension required, as

we have calculated, is 98,720 pounds. It would, therefore, seem that the bolt used is too small. The following excuses may be extended, however. In the first place the wind was considered as acting all the way down the stack, then the tension on the bolt was considered as acting on the circumference of the stack instead of about nine inches from it, then the tension due to the wind was considered uniform all around the base of the stack, and lastly, the concrete on the inside of the stack base, which holds down the structure has been neglected. If I had my way, I should have used about three inch bolts, but I do not doubt for one minute but that the ones used will be strong enough.

We now come to the design of the foundation itself. Unfortunately I did not figure out this foundation. I assisted in making the first design of the base. This, however, had to be changed entirely on account of water pipes, and to fit in with other foundations; also the character of the soil,

which at first was thought to be of sand was found to be of firmer material. I will content myself, therefore, with simply investigating its stability as it is built. Now as long as the resultant of the wind and load forces fall within the middle third of the base, there can be no tension on the concrete. The foundation bolts run down well into the base, and we can assume that they take all the tension as far as the anchor plates. We have, therefore, only to see if there is any tension between the foundation and the earth on the windward side of the stack, and if the compression on the other side is within the safe limit of bearing power of the earth. This bearing power is assumed to be 2 1/2 tons per square foot, the weight of the earth resting upon the different steps of the foundation is neglected, although some engineers take it into account. The total weight of the stack with the lining is 682,176 pounds, the weight of the foundation is 1,485,405 pounds, the total weight is, there-

fore, 2,167,581 pounds. The area of the base is 784 square feet, the pressure per square foot due to weight is, therefore, 2,764.6 pounds. Now the height of the center of wind pressure above the base is found by the formula,

$$S = \frac{D + 2D'}{D + D'} \times \frac{H}{3}$$

where D = diameter of shell at floor in feet = 13.30

Where D' = diameter of shell at top in feet = 9.93

where H = Height of stack in feet = 225.00

Therefore S = 107.6 feet.

The projected area of the stack is,

$(D + D')H \times 1/2 = 2,613.4$ square feet

Therefore the moment tending to overturn the stack in foot pounds is,

$2,613.4 \times 25 \times (107.6 + 16) = 8,035,000$ foot pounds = sI/c.

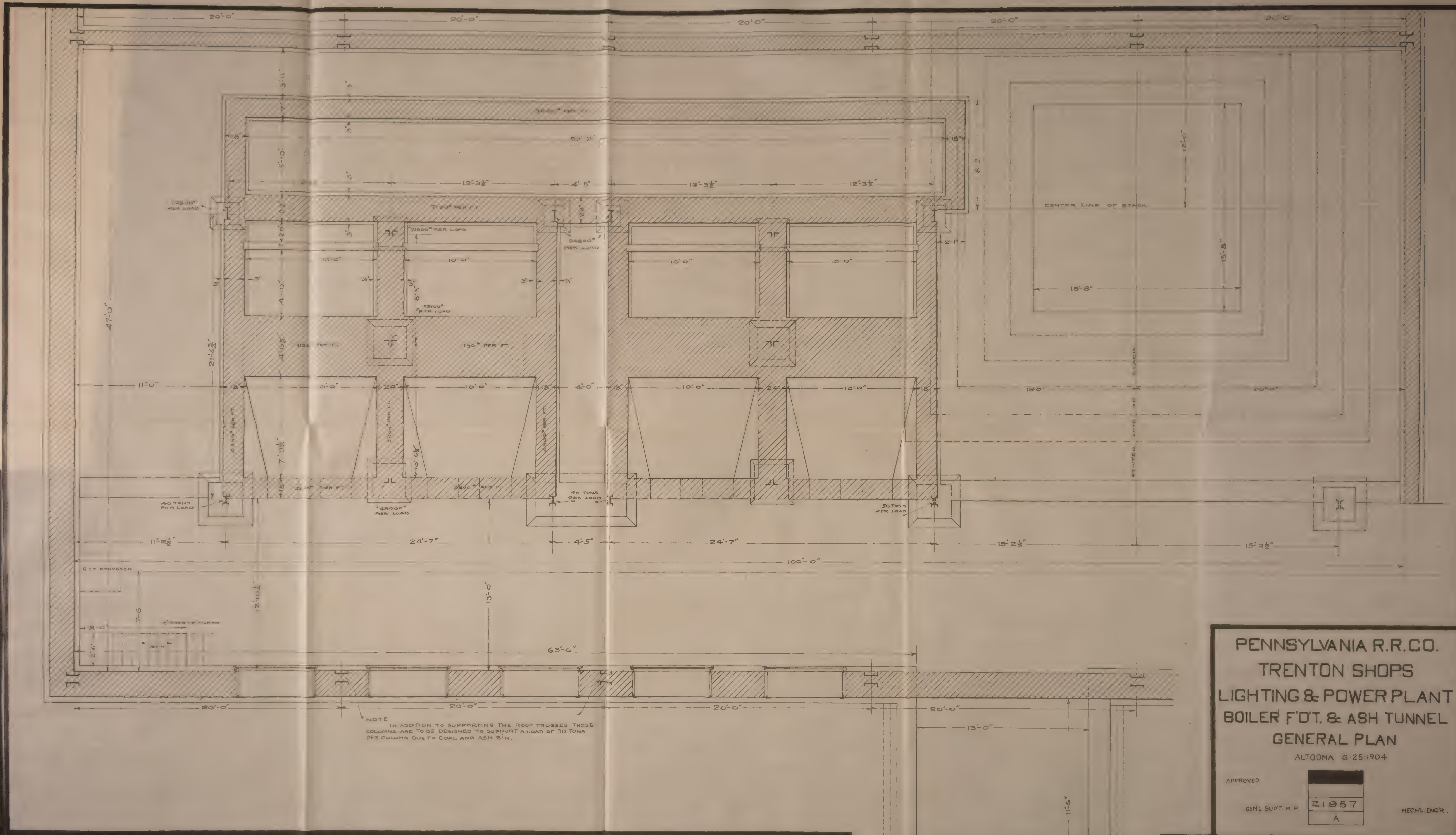
where s = pressure or tension per square foot,

where I/c = section modulus of base area, in feet cubed. Therefore s equals 9,035,000 divided by 3658.7 or 2,196 pounds. Therefore the maximum compression

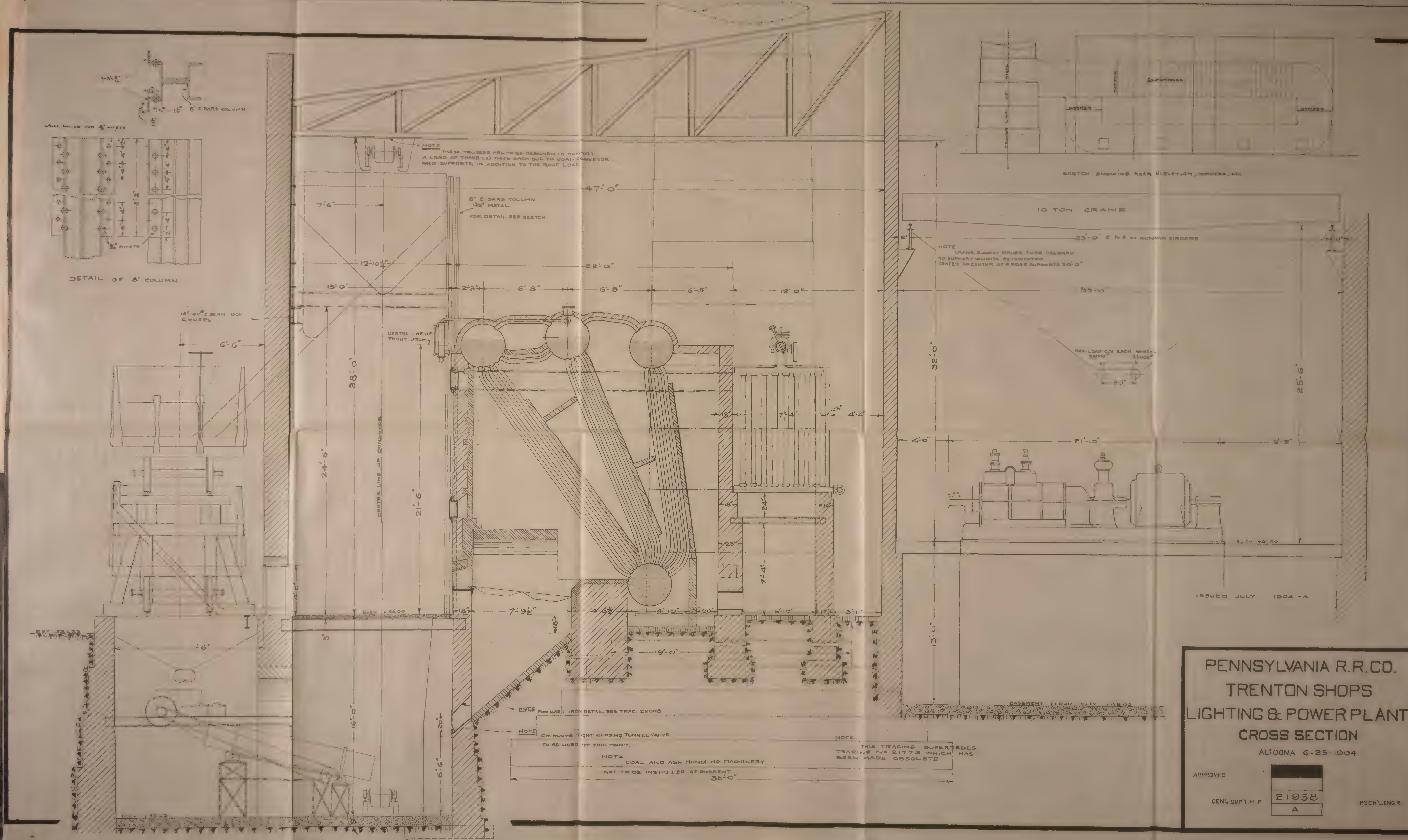
is 2,196 + 2,765 = 4,961 pounds per square foot, or less than 2 1/2 tons, which is the safe bearing power allowed for the soil. Also opposite this compression will be 2,765 - 2,196, or 569 pounds per square foot, whoch shows that the resultant of the forces acts within the middle third of the base.

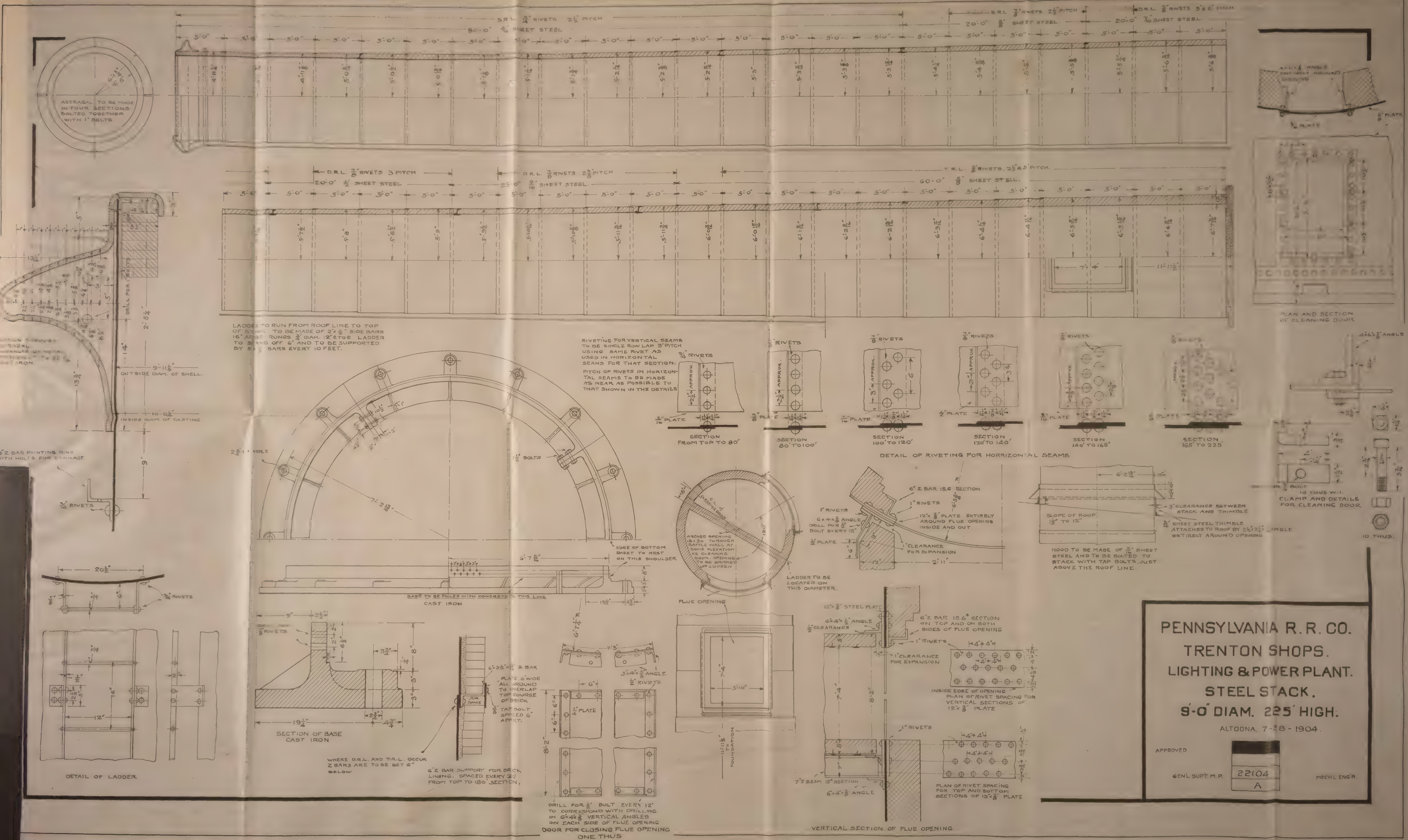
Time is too short to describe the strength of the ladder construction, the astrigal and the base ring. They were, however, all taken from tried out designs, and are, therefore, considered safe, as shown in the drawings. The Z bars where the joints are double riveted are held by a single row of 7/8 inch rivets on a 2 1/2" pitch. This as can be seen from the curve diagram, is more than safe. It was however used to make this riveting correspond with the riveting just above it.

I attach hereto prints of the stack, its location, its foundation, and the curve showing its strength.

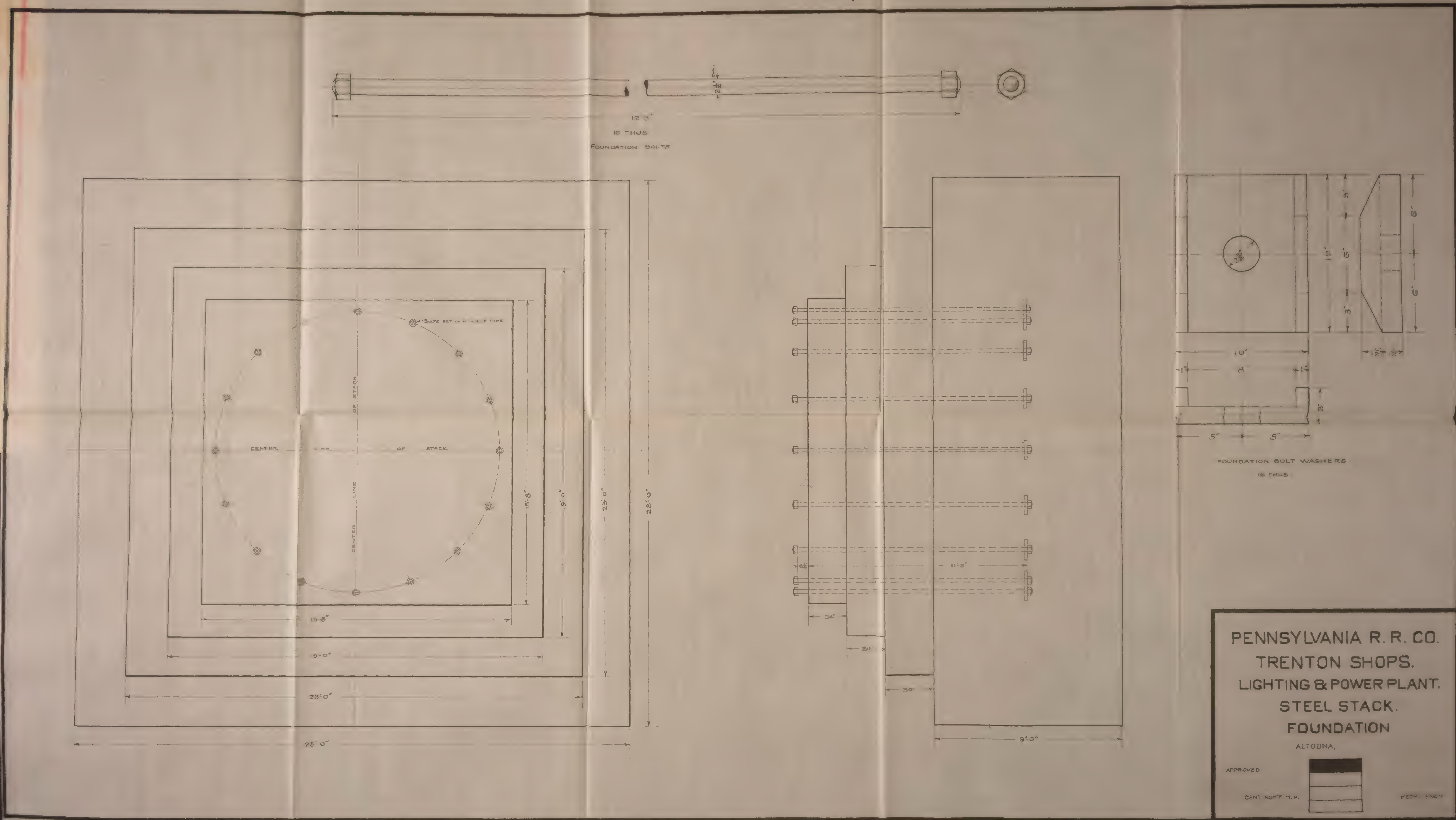


PENNSYLVANIA R.R.CO.
TRENTON SHOPS
LIGHTING & POWER PLANT
BOILER F'D'T. & ASH TUNNEL
GENERAL PLAN
ALTOONA 6-25-1904
APPROVED
GEN'L SUPT. M. P.
21957
A
MECH'L ENG'R.
NOTE
IN ADDITION TO SUPPORTING THE ROOF TRUSSES THESE COLUMNS ARE TO BE DESIGNED TO SUPPORT A LOAD OF 30 TONS PER COLUMN DUE TO COAL AND ASH BIN.
CENTER LINE OF STACK
100'-0"
63'-6"
24'-7"
4'-5"
15'-2½"
11'-2½"
47'-0"
40 TONS PIER LOAD
50 TONS PIER LOAD
45000# PIER LOAD
24800# PIER LOAD
21500# PIER LOAD
7100# PER FT.
1150# PER FT.
15'-8"
15'-0"


PENNSYLVANIA R.R.CO.
TRENTON SHOPS
LIGHTING & POWER PLANT
CROSS SECTION
ALTOONA 6-25-1904
APPROVED
GEN'L SUP'T M.P.
21958
A
MECH'L ENGR.
ISSUED JULY 1904 - A
DETAIL OF 8" COLUMN
8" Z BARS COLUMN
DRILL HOLES FOR 3/4" RIVETS
NOTE
THESE TRUSSES ARE TO BE DESIGNED TO SUPPORT
A LOAD OF THREE (3) TONS EACH DUE TO COAL CONVEYOR
AND SUPPORTS, IN ADDITION TO THE ROOF LOAD
8" Z BARS COLUMN
3/8" METAL
FOR DETAIL SEE SKETCH
SKETCH SHOWING REAR ELEVATION, DAMPERS ETC.
DAMPER
10 TON CRANE
35'-0" ℄ to ℄ of RUNWAY GIRDERS
NOTE
CRANE RUNWAY GIRDER TO BE DESIGNED
TO SUPPORT WEIGHTS AS INDICATED
CENTER TO CENTER OF GIRDER SUPPORTS 20'-0"
MAX. LOAD ON EACH WHEEL
15"-42# I BEAM BOX GIRDERS
CENTRE LINE OF FRONT DRUM
CENTER LINE OF CONVEYOR
47'-0"
22'-0"
13'-0"
2'-3"
6'-8"
6'-8"
6'-5"
12'-0"
7'-6"
12'-10 1/2"
6'-6"
38'-0"
24'-6"
21'-6"
32'-0"
25'-6"
13'-0"
16'-0"
6'-6"
7'-4"
4'-4"
4'-0"
21'-10"
9'-2"
7'-9 1/2"
4'-6 1/2"
4'-10"
5'-10"
19'-0"
11'-6"
ELEV. +55.00
ELEV. +53.625
ELEV. +61.00
BASEMENT FLOOR ELEV. +48.00
NOTE FOR CAST IRON DETAIL SEE TRAC. 22008
NOTE C.W. HUNTS TIGHT CLOSING TUNNEL VALVE
TO BE USED AT THIS POINT.
NOTE
COAL AND ASH HANDLING MACHINERY
NOT TO BE INSTALLED AT PRESENT
35'-0"
NOTE
THIS TRACING SUPERSEDES
TRACING No 21773 WHICH HAS
BEEN MADE OBSOLETE



PENNSYLVANIA R. R. CO.
TRENTON SHOPS.
LIGHTING & POWER PLANT.
STEEL STACK.
9-0" DIAM. 225' HIGH.
ALTOONA, 7-28-1904.
APPROVED
GEN'L SUPT M.P.
MECH'L ENG'R.
22104
A
ASTRAGAL TO BE MADE IN FOUR SECTIONS BOLTED TOGETHER WITH 1" BOLTS
OUTSIDE DIAM. OF SHELL.
INSIDE DIAM. OF CASTING
3"Z BAR PAINTING RING WITH HOLES FOR DRAINAGE
RIVETS
LADDER TO RUN FROM ROOF LINE TO TOP OF STACK. TO BE MADE OF 2"x½" SIDE BARS 16" APART. RUNGS ¾" DIAM. 12" C TO C. LADDER TO STAND OFF 6" AND TO BE SUPPORTED BY BARS EVERY 10 FEET.
RIVETING FOR VERTICAL SEAMS TO BE SINGLE ROW LAP 3" PITCH USING SAME RIVET AS USED IN HORIZONTAL SEAMS FOR THAT SECTION. PITCH OF RIVETS IN HORIZONTAL SEAMS TO BE MADE AS NEAR AS POSSIBLE TO THAT SHOWN IN THE DETAILS.
SECTION FROM TOP TO 80'
SECTION 80' TO 100'
SECTION 100' TO 120'
SECTION 120' TO 140'
SECTION 140' TO 165'
SECTION 165' TO 225'
DETAIL OF RIVETING FOR HORRIZONTAL SEAMS
PLAN AND SECTION OF CLEANING DOOR
CLAMP AND DETAILS FOR CLEANING DOOR
10 THUS.
EDGE OF BOTTOM SHEET TO REST ON THIS SHOULDER
BASE TO BE FILLED WITH CONCRETE TO THIS LINE
CAST IRON
1½" BOLTS
FLUE OPENING
LADDER TO BE LOCATED ON THIS DIAMETER.
6"Z BAR 15.6" SECTION
12"x⅜" PLATE ENTIRELY AROUND FLUE OPENING INSIDE AND OUT
1" CLEARANCE FOR EXPANSION
SLOPE OF ROOF 1½" TO 12"
3" CLEARANCE BETWEEN STACK AND THIMBLE
SHEET STEEL THIMBLE ATTACHED TO ROOF BY ANGLE ENTIRELY AROUND OPENING
HOOD TO BE MADE OF SHEET STEEL AND TO BE BOLTED TO STACK WITH TAP BOLTS JUST ABOVE THE ROOF LINE.
SECTION OF BASE CAST IRON
DETAIL OF LADDER
WHERE D.R.L. AND T.R.L. OCCUR Z BARS ARE TO BE SET 6" BELOW
6"Z BAR SUPPORT FOR BRICK LINING. SPACED EVERY 20' FROM TOP TO 180' SECTION.
DRILL FOR ½" BOLT EVERY 12" TO CORRESPOND WITH DRILLING IN 6"x4"x⅜" VERTICAL ANGLES ON EACH SIDE OF FLUE OPENING
DOOR FOR CLOSING FLUE OPENING
ONE THUS
6"Z BAR 15.6" SECTION ON TOP AND ON BOTH SIDES OF FLUE OPENING
INSIDE EDGE OF OPENING
PLAN OF RIVET SPACING FOR VERTICAL SECTIONS OF 12"x⅜" PLATE
PLAN OF RIVET SPACING FOR TOP AND BOTTOM SECTIONS OF 12"x⅜" PLATE
7"I BEAM 15" SECTION
VERTICAL SECTION OF FLUE OPENING

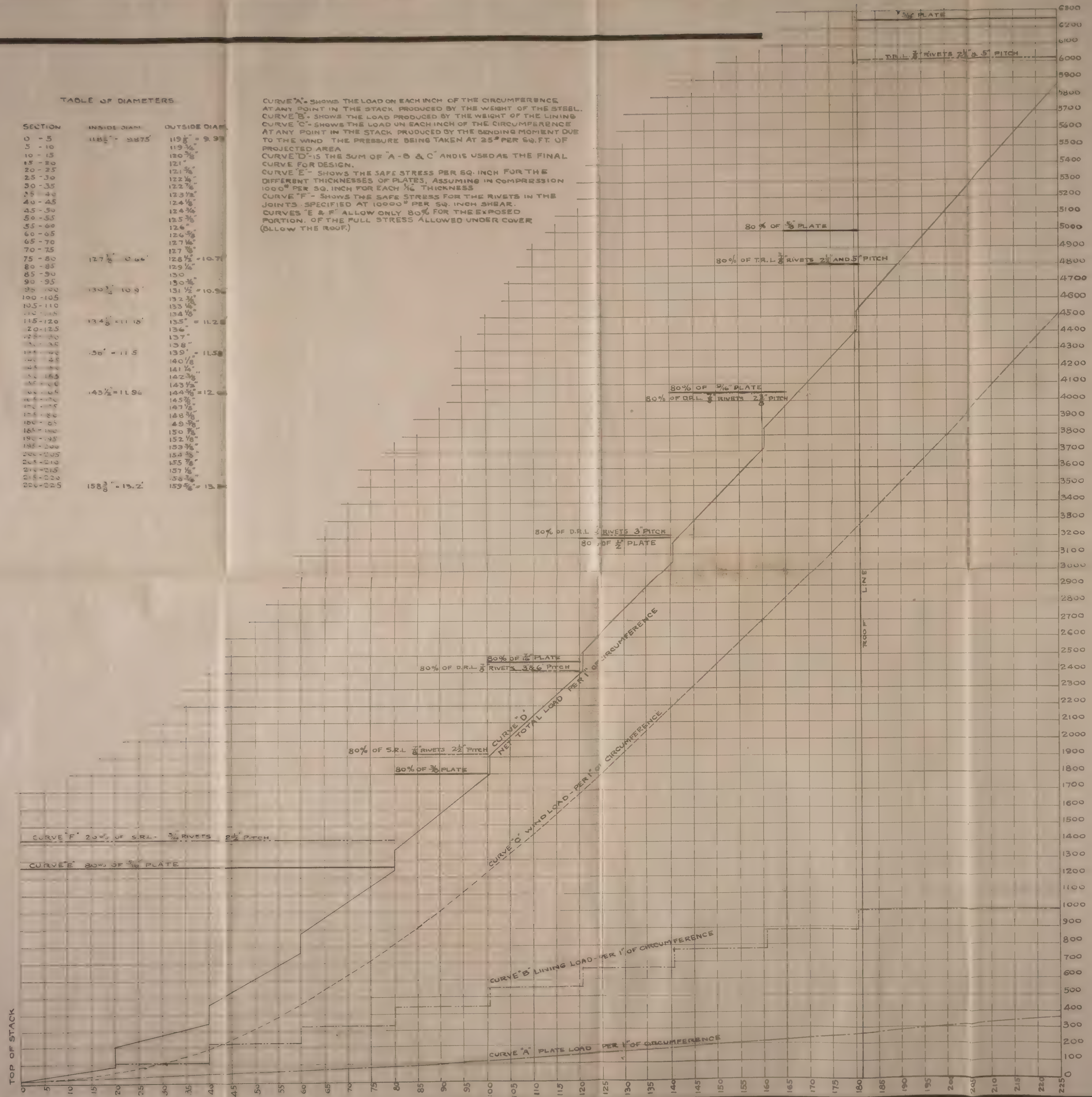


12' 3"
16 THUS
FOUNDATION BOLTS
BOLTS SET IN 3" W.I. PIPE
CENTER LINE OF STACK
CENTER LINE OF STACK
15'-8"
19'-0"
23'-0"
28'-0"
11'-5"
54"
24"
30"
9'-8"
10"
8"
1"
1"
3"
6"
12"
6"
6"
3"
5"
5"
FOUNDATION BOLT WASHERS
16 THUS
PENNSYLVANIA R. R. CO.
TRENTON SHOPS.
LIGHTING & POWER PLANT.
STEEL STACK.
FOUNDATION
ALTOONA,
APPROVED
GEN'L SUP'T. M. P.
MECH'L ENG'R

TABLE OF DIAMETERS

SECTION	INSIDE DIAM.	OUTSIDE DIAM.
0 - 5	118⅝" = 9.875'	119⅝" = 9.9
5 - 10		119¾"
10 - 15		120⅜"
15 - 20		121"
20 - 25		121⅝"
25 - 30		122¼"
30 - 35		122⅞"
35 - 40		123½"
40 - 45		124⅛"
45 - 50		124¾"
50 - 55		125⅜"
55 - 60		126"
60 - 65		126⅝"
65 - 70		127¼"
70 - 75		127⅞"
75 - 80	127⅞" = 10.66'	128½" = 10.7
80 - 85		129¼"
85 - 90		130
90 - 95		130¾"
95 - 100	130¾" = 10.9'	131½" = 10.9
100 - 105		132⅜"
105 - 110		133¼"
110 - 115		134⅛"
115 - 120	134⅛" = 11.18'	135" = 11.2
120 - 125		136"
125 - 130		137"
130 - 135		138"
135 - 140	138" = 11.5	139" = 11.58
140 - 145		140⅛"
145 - 150		141¼"
150 - 155		142⅜"
155 - 160		143½"
160 - 165	143½" = 11.96	144⅝" = 12.
165 - 170		145⅞"
170 - 175		147⅛"
175 - 180		148⅜"
180 - 185		149⅝"
185 - 190		150⅞"
190 - 195		152⅛"
195 - 200		153⅜"
200 - 205		154⅝"
205 - 210		155⅞"
210 - 215		157⅛"
215 - 220		158⅜"
220 - 225	158⅜" = 13.2'	159⅝" = 13.

CURVE "A" - SHOWS THE LOAD ON EACH INCH OF THE CIRCUMFERENCE AT ANY POINT IN THE STACK PRODUCED BY THE WEIGHT OF THE STEEL.
CURVE "B" - SHOWS THE LOAD PRODUCED BY THE WEIGHT OF THE LINING
CURVE "C" - SHOWS THE LOAD ON EACH INCH OF THE CIRCUMFERENCE AT ANY POINT IN THE STACK PRODUCED BY THE BENDING MOMENT DUE TO THE WIND THE PRESSURE BEING TAKEN AT 25# PER SQ. FT. OF PROJECTED AREA
CURVE "D" - IS THE SUM OF "A - B & C" AND IS USED AS THE FINAL CURVE FOR DESIGN.
CURVE "E" - SHOWS THE SAFE STRESS PER SQ. INCH FOR THE DIFFERENT THICKNESSES OF PLATES, ASSUMING IN COMPRESSION 1000# PER SQ. INCH FOR EACH 1/16" THICKNESS
CURVE "F" - SHOWS THE SAFE STRESS FOR THE RIVETS IN THE JOINTS SPECIFIED AT 10000# PER SQ. INCH SHEAR.
CURVES "E & F" ALLOW ONLY 80% FOR THE EXPOSED PORTION. OF THE FULL STRESS ALLOWED UNDER COVER (BLLOW THE ROOF.)



PENNSYLVANIA R. R. CO.
TRENTON SHOPS.
LIGHTING & POWER PLANT.
STEEL STACK.
STRESS CURVES.

ALTOONA,

APPROVED

GEN'L SUP'T. M. P. | MECH'L ENGR.

A